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Stress changes associated with driving pile groups in clayey silt

Des modifications d'effort liées à l'enfoncement des groupes de pieux dans la vase argileuse

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ABSTRACT : Measurements obtained from lateral total stress sensors located on the centre pile of a small driven pile group and pneumatic piezometers installed in the vicinity of the group are used to assess the disturbance associated with installation of driven piles in a clayey silt. The observations made are substantiated in a subsequent load test on one pile in the group.

RESUME : Les dimensions obtenues des capteurs d'effort latérals situés sur le pieu central d'un petit groupe de pieux enfoncés et des piézomètres pneumatiques installés à proximité du groupe sont employees pour évaluer la perturbation liée à l'installation des pieux enfoncés dans une vase argileuse. Les observations faites sont justifiées dans un essai de charge ultérieur sur un pieu dans le groupe.

1 INTRODUCTION

It is well established that the working performance of any driven pile is influenced by the new effective stress regime set up by its installation. Additional effective stress changes will also arise due to the driving of neighbouring piles within a group and the extent of these will depend, amongst other factors, on the pile spacing, the number of piles and the soil type. Despite some recent significant theoretical advances, considerable uncertainty remains regarding the reliability of numerical approaches attempting to predict stress changes associated with single pile driving; the prediction of stress changes associated with driven pile group installation are even less certain. Experimental measurements therefore fulfil a vital role in improving our understanding of the mechanisms involved and their scarcity instigated a programme of full-scale pile load tests on instrumented single piles and small groups of five piles at a soft clay-silt site near Belfast, Northern Ireland. In addition to the load tests, measurements of excess pore water pressure (in the soil around a group) and total horizontal stresses (on the shaft of the centre pile of a group) were recorded during pile installation and over the period required for near-full equalization/consolidation of the soil. This paper reports and examines these data with a view to offering some insights into the installation process and the relative effects of group installation and interaction under load.

2 SOIL CONDITIONS

A site on the shores of Belfast Lough (10km north east of Belfast city centre) comprising a 7m layer of soft estuarine clayey silt provided the location for an experimental investigation into the performance of vertically loaded single piles and pile groups carried out by Trinity College Dublin (TCD). Laboratory tests on piston samples and a large range of in-situ tests have been performed by TCD for which full details are provided by McCabe (2002). The strata of primary geotechnical interest (between ≈1.0m and ≈8.5m depth) are locally referred to as *sleech;* Figure 1 provides a summary of some classification data.

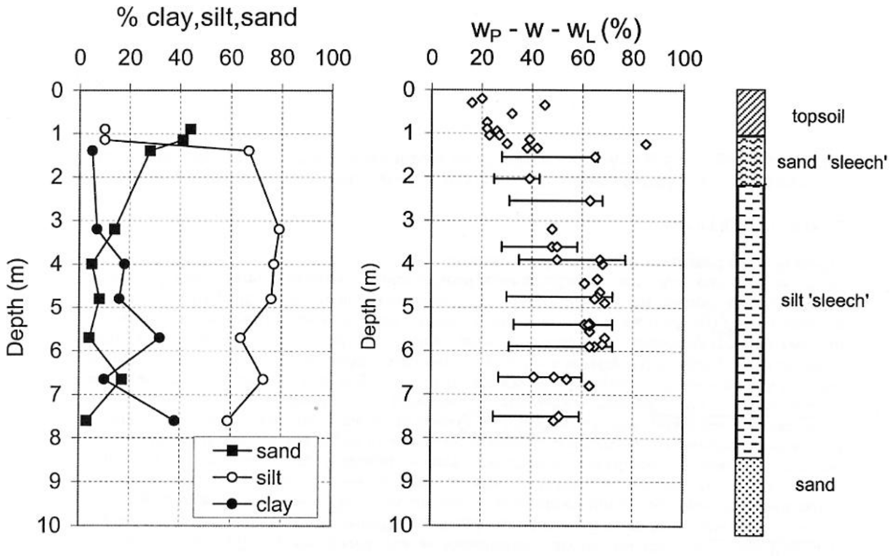

Figure 1. Stratigraphy at the Belfast test site

Apart from a sandier horizon at between about 1 and 1.8m depth, the '*sleech'* is a soft organic clayey silt, which extends to a depth of ≈8.5m. The material's high plasticity (I_p ≈ 35%) is partly due to the nature of the organics present (organic content =11 ±1%) and I_p falls to between 15% and 20% when these organics are removed. X-ray diffraction and chemical analyses indicate that illite, chlorite and quartz make up the clay fraction. The *sleech* has a typical (uncorrected) vane strength of ≈ 20-25 kPa and its vertical yield stress ratio (YSR; determined in oedometer tests) reduces from ≈1.5 at 3m to ≈1.1 below 6m. The material has a high constant volume friction angle (ϕ'_{cv}) of ≈33°.

3 EXPERIMENTAL PROGRAMME

The results presented in this paper relate to some of the data obtained from three driven pile groups, designated *PG1*, *PG2* and *PG3*. *PG1* and *PG2* were load-tested to ultimate conditions in compression and tension respectively in subsequent load tests (and presented in McCabe 2002), while one pile in *PG3* was load tested alone in compression. All piles employed were 250mm square (B=0.25m) precast units that were essentially 'pushed' to their final penetration depth (L) of 6m under the weight of a 2 tonne driving hammer.

As shown in the insert to Figure 2, pile group configurations comprised a centre pile (designated *G3*) surrounded by four corner piles (designated *G1*,*G2*,*G4* & *G5*); the side width of the pile groups was ≈1.2m and such that the spacing between *G3* and corner piles averaged at ≈2.8B. Installation of the centre pile was followed by installation of corner piles, *G1*, *G4*, *G5* and *G2*. *G3* was finally re-tapped to reverse the observed uplifts of up to ≈5mm caused by the corner pile installations.

Two total horizontal stress sensors (of the oil-filled flat jack variety) were cast flush with one face of pile *G3* in group *PG1* and were located at normalized heights above the pile's base, h/B, of 3 and

11. When pile *G3* had been installed to 6m, these sensors were at depths of 3.25m and 5.25m. The sensors were monitored continuously over the time required for group installation and intermittently until load testing. Since *G3* was the first of the five piles to be driven, the data recorded before the second pile (*G1*) was driven correspond to those of a single pile.

A total of nine piezometers were pushed a distance of ≈1m below the base of pre-augered holes in the vicinity of the proposed location of *PG2*; the holes were then reinstated with a cement/bentonite grout. Installation of *PG2* followed one month later at which point pore pressure readings were recorded manually as often as possible using a pneumatic readout unit. Some piezometers were difficult to access due to the position of the piling rig, so maximum excess pore pressures may not have been captured in all cases.

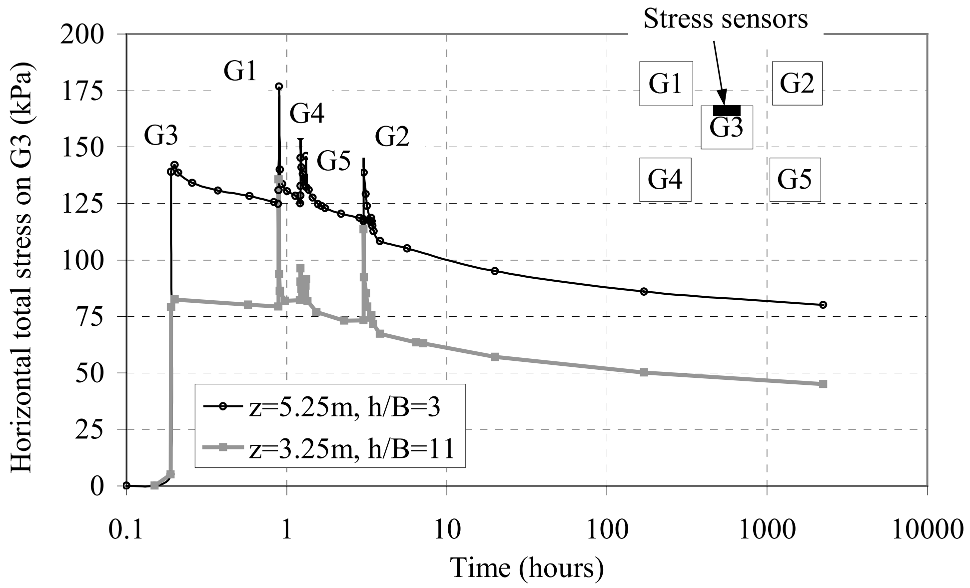

Figure 2. Lateral stress variations on centre of pile group during installation

4 INSTALLATION

Variations in horizontal total stresses (σ_h) measured on *G3* during and after installation of PG1 are shown on Figure 2. Clearly, greatest individual increases in σ_h occur as a result of G3's own installation. The total stress magnitudes may be represented more generally in the form of a total stress ratio, defined as H $[= (\sigma_h - u_0)/\sigma'_{vo}]$, where u_0 is the ambient (hydrostatic) pore pressure and σ'_{v0} is the free field vertical effective stress. The H_i ratios (H during installation) obtained in this manner for pile *G3*'s installation of 2.13 at h/B=3 and 1.74 at h/B=11 are typical of single piles driven in lightly overconsolidated clay and fall within the bounds of a H_i database summarized by Lehane et al. (1994).

The subsequent installation of each of the four corner piles generates further smaller increases in σ_h on the shaft of *G3*. However, unlike the centre pile's own installation, each of these increases in σ_h is short-lived and stresses quickly return to values similar to what one might expect after single pile installation. The actual increases in σ_h occur first at the sensor at h/B=11; these increases then diminish as the pile penetrates deeper and causes the stresses at the sensor located at h/B=3 to increase.

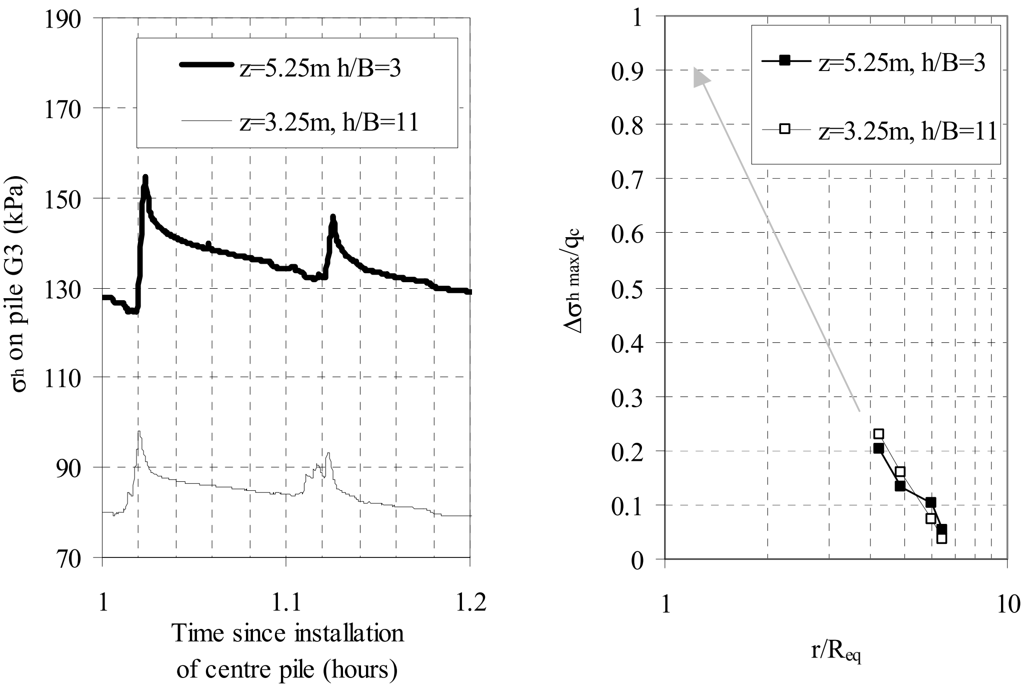

Figure 3. (a) Lateral stress changes on G3 during installation of G4 & G5, (b) Normalised maximum lateral stress changes associated with corner pile installations.

A typical example of this trend is indicated on Figure 3(a) during installation of piles *G4* and *G5*. Given the relatively rapid decay in σ_h as the tip of the pile passes beneath the instrument sensor, it may be surmised that the observed increases in σ_h are associated with increases in total stress level near the tips of the corner piles during their installation. The relative increases in σ_h during a given corner pile installation vary with the distance of the sensor from that location. This trend is depicted on Figure 3(b) which plots the maximum recorded increase in σ_h (=$\Delta\sigma_{hmax}$) following each corner pile installation against the distance (in plan) of the sensor from the axis of the corner pile (*r*). The values of $\Delta\sigma_{hmax}$ are normalized by the CPT end resistance ($q_c \approx$200 kPa) and the radial distances from the sensor are normalized by the equivalent radius of the piles (R_{eq}=B/√π). The observed reduction in $\Delta\sigma_{hmax}$ with log *r* is reminiscent of stress changes associated with a cavity expansion process. As seen on Figure 2, by the end of group installation, it is evident that the effect of installing the corner piles has not caused an accumulation of σ_h at the centre pile shaft. The peak values of H_i recorded over the period of group installation (denoted $H_{i,g}$) are 3.3 and 2.9 at h/B=3 and 11 respectively.

The maximum excess pore pressure ratios ($\Delta u_{max}/\sigma'_{vo}$) recorded by pneumatic piezometers located at depths between 2 m and 5.5 m during installation of group *PG2* are plotted on Figure 4(a) against their relative distance from the group centre (*r*). It is evident that $\Delta u_{max}/\sigma'_{vo}$ reduces approximately linearly with the logarithm of r/R_{eq}. Although no piezometers were located within the group perimeter, the value of $\Delta u_{max}/\sigma'_{v0}$ at r =R_{eq} (i.e. at the shaft of the centre pile, *G3*) may be estimated by assuming that the horizontal effective stress acting during installation on *G3* is no larger than ≈0.2σ'_{v0} (e.g. see Lehane et al. 1994). The value of $\Delta u_{max}/\sigma'_{v0}$ is therefore expected to be approximately ($H_{i,g}$ – 0.2), which is ≈2.7 at h/B=11. The radial variation of $\Delta u_{max}/\sigma'_{v0}$ for installation of group *PG2* is compared with maximum excess pore pressure ratios recorded during single driven pile installation in St Alban Clay with YSR ≈2.2 (Roy et al. 1981) and Young Bay Mud with YSR ≈1.3 (Pestana et al. 2002).

Bearing in mind that $\Delta u_{max}/\sigma'_{v0}$ varies approximately with $YSR^{0.36}$ (Lehane et al. 1994, Lehane 1992), it may be inferred from Figure 4(a) that slightly higher excess pore pressure ratios surround the pile group and that these extend to a larger radial distance. The extent of the excess pore pressure field around a pile group clearly depends on the size of the group. For example, Bozozuk et al. (1978) present data for a group containing 116 piles (with $s/R_{eq}=10$) and report a relatively constant and significant excess pore pressure ratio extending from the group edge to a distance of $30R_{eq}$ beyond this edge. The decay of $\Delta u_{max}/\sigma'_{v0}$ with increasing *r* is also less pronounced than that observed in Belfast for a group of 13 piles with $s/R_{eq}=8$, reported by Fellenius & Samson (1976). Thus, while the excess pore pressures generated in the vicinity of a given pile in a group due to installation of a neighbouring pile are restricted by the fact that the soil is at the critical state condition (and hence at constant mean effective stress), excess pore pressures outside of the 'plastic zone' may accumulate with the addition of each pile. The net result is that the excess pore pressure field associated with a large group extends significantly further than the corresponding field around a single pile.

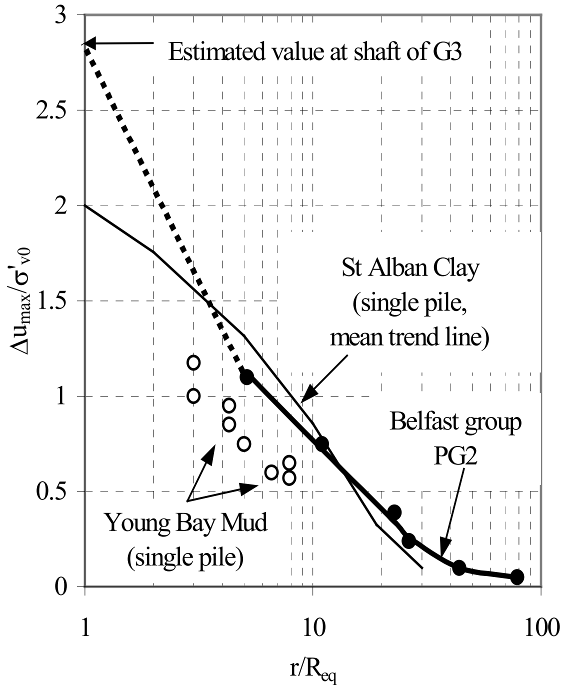

Figure 4. Radial variation of maximum excess pore pressures

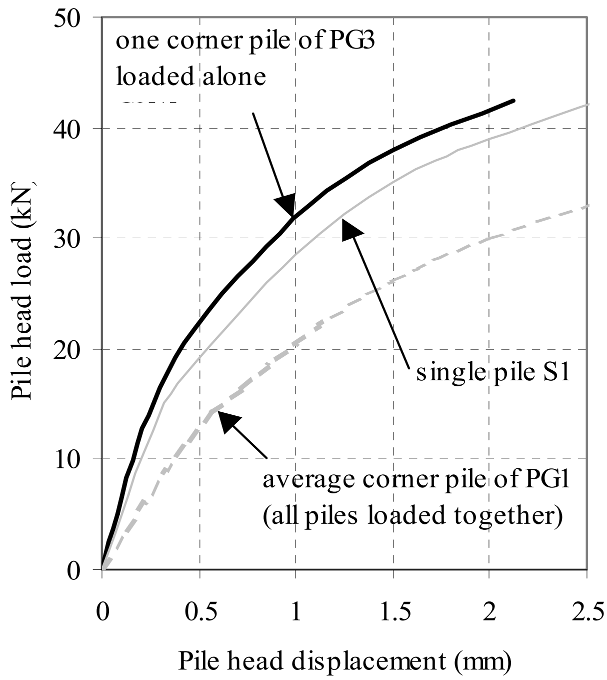

Figure 5. Response of corner pile in load tests

5 EQUALIZATION

The σ_h data on Figure 2 suggest that equalization of the soil at the centre pile shaft (G3) was almost completed after three months (≈2000 hours). The lateral stress coefficients at this time, H_c, of 0.81 (at $h/B=3$) and 0.57 (at $h/B=11$) may be compared with the estimated initial stress coefficient (i.e. K_o) of between 0.5 and 0.6. Drawing on comparable equalization data for a single driven pile in Bothkennar clay (Lehane & Jardine 1994), McCabe (2002) shows that the equalized lateral effective stresses on *G3* are likely to be very similar to (or up to ≈15% smaller than) those on a comparable isolated pile. The effects of driving adjacent piles at $s/B=2.8$ on a pile's performance are therefore seen not to be significant.

The degree of dissipation indicated by the pneumatic piezometers, all of which were located at $r/R_{eq} > 5$, was slower than that typically observed at the shaft of a driven pile, but is consistent with expectations for radial drainage; see McCabe (2002). Similarly slower rates of dissipation are reported for $r/R_{eq} > 1$ by Roy et. al. (1981) around a single pile in Champlain clay.

6 LOAD TESTING

A static maintained load compression test was performed on one corner pile (*G2*) on pile group (*PG3*). The observed pile head load displacement curve of *G2* is compared on Figure 5 with pile head load-displacement curves obtained for (i) a single isolated pile (*S1*) and (ii) with the average corner pile response observed during the compression group load test on *PG1*. The curves indicated on Figure 5 correspond to a pile displacement rate of 0.004 mm/minute.

The similarity between the load-displacement responses of pile *G2* in group *PG3* and the single pile (*S1*) suggest that adjacent pile installations have minimal effect on the shear stiffness of the ground at any group pile shaft, and that group installation effects should not dictate the subsequent performance of the group under load. This finding is compatible with the trends indicated by the lateral stress measurements. The load displacement curve for a corner pile observed during the compression group test is in sharp contrast to the other two load-displacement curves, indicating the importance that interaction under load plays in determining the stiffness and ultimate capacity of group piles.

7 CONCLUSION

The Belfast pile experiments have shown driving adjacent piles at s/B=2.8 around a given pile in a soft clayey silt does not significantly alter the long term effective stress regime around the pile.

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